MEMORANDUM REPORT

Granite Reef Diversion Dam Hydraulic Model Study

by

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for

Salt River Project
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Introduction

Background

Granite Reef Diversion Dam (Figure 1) was constructed from 1906 to 1908 by the Bureau of Reclamation. The dam, part of the Salt River Project (SRP), is located on the Salt River 4 miles downstream from the confluence of the Salt and Verde Rivers, east of Phoenix, Arizona. The dam diverts water from upstream storage reservoirs to the irrigation canal system.

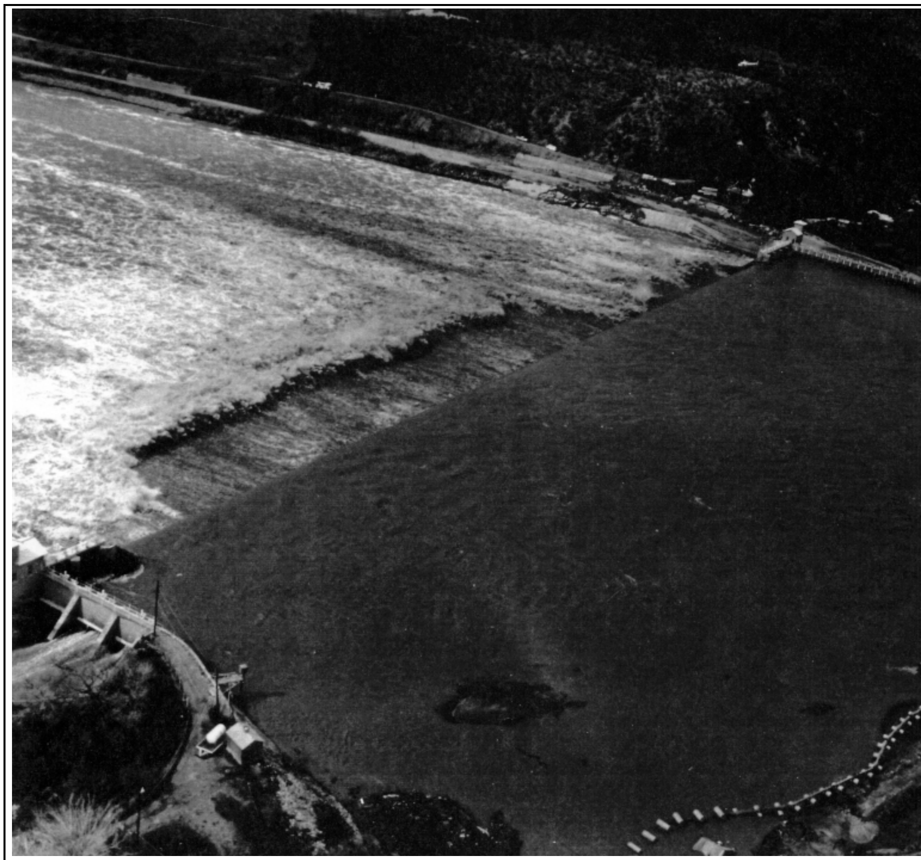


Figure 1. Granite Reef Diversion Dam. *Unknown date and discharge. Canal headworks can be seen on the north side of the river (upper-right) and south side (lower-left). In the center of the spillway (upper-left), the hydraulic jump is not retained on the downstream apron. Granite outcroppings on the north and south sides of the downstream apron retain the hydraulic jump to near the base of the dam*

The 20-ft-high diversion dam (Figure 2) consists of a 1,000-ft-long ogee-shaped crest across the river at El. 1310 ft, with a 75-ft-long downstream apron at El. 1290 ft. Canal headworks and sluiceways are present on both the north and south sides of the dam. The sluiceway gate sills are located at El. 1302 ft with four gates on the north sluiceway and two gates on the south sluiceway. The canal head gate sills, located at El. 1306 ft, provide diversion capacities of 2,000 and 1,600 ft^3/s for the north and south canals, respectively.

Historically, erosion damage has occurred in the river immediately downstream of the diversion dam and both the north and south sluiceways. Figure 1 displays typical behavior of the hydraulic jump, which sweeps off most of the downstream apron and is held back by the outcropping of granite rock at the north and south ends of the apron. Recent erosion has caused a scour hole at the end of the existing apron estimated to be more than 20 ft deep resulting in apron sections and parts of the 12-ft-deep by 4-ft-wide cutoff wall washing out and requiring replacement.

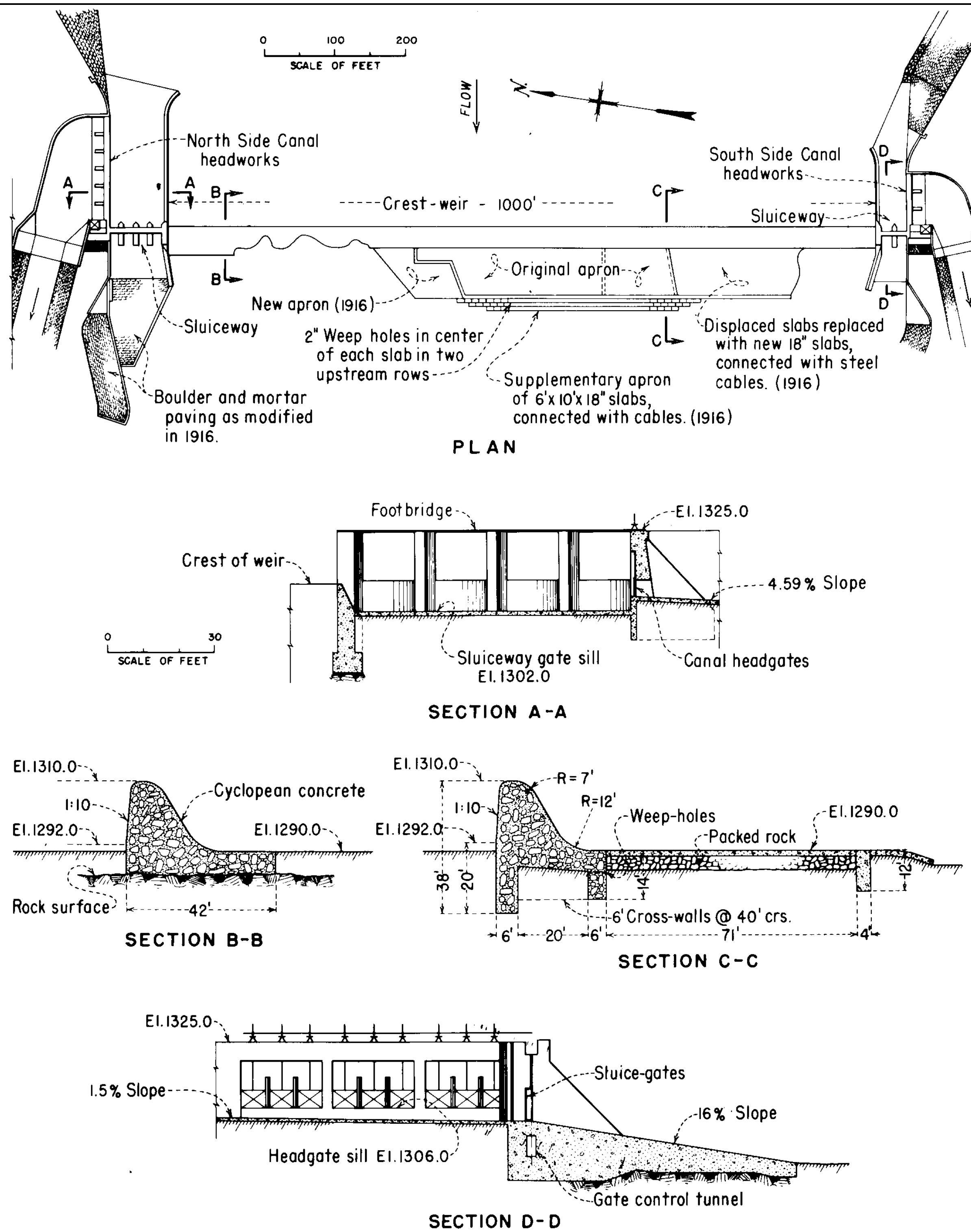


Figure 2. Plan and sections of Granite Reef Diversion Dam. *The plan view and section C-C does not show historical and current damage or replacement of the 18 inch thick apron with 6- to 8-ft mass concrete. Most of the supplementary apron is currently washed out.*

Purpose of Study

The purpose of this model study was to:

- determine the stage-discharge characteristics of the ogee crest with the forebay silted to El. 1307 ft (partially silted) and 1310 ft (fully silted),
- determine the spillway apron length at the existing elevation that will retain the hydraulic jump for various flows,
- design and test sills that will afflictively convert the existing apron to an energy dissipater that will force the hydraulic jump on the existing apron and reduce erosion downstream,
- design and test a stepped spillway overlay to dissipate energy and to retain the hydraulic jump on the existing apron, and
- measure velocities in the modeled river channel downstream from the recommended alternative to determine the need for additional channel protection.

Conclusions

Stage-discharge relationships were developed for two levels of sediment upstream of the dam. The relationships were developed for the south stilling well location. The relationships show that the discharge decreases as the sediment level increases for a given head (Figure 5). The stage-discharge measured at the south stilling well could be negatively affected by standing waves caused by shallow depths and deposition of sediment. The stage-discharge relationship presented in this report should provide values that are in the middle of the ranges that will likely be encountered.

Extending the existing apron horizontally to any reasonable length proved inadequate to retain the hydraulic jump because the depth of tailwater is inadequate. The spillway apron length required to retain a hydraulic jump on apron was greater than 180 ft, which is less economically viable than other alternatives studied. Also, sensitivity to changes in tailwater, caused by further degradation of the river channel, would remain.

Several sills and secondary stilling basin configurations were tested to force a hydraulic jump on the existing apron, and to reduce erosion downstream. Configuration No. 7 shown in Figure 16 is the recommended option. The 5-ft-high sill is located 50 ft upstream from the end of the existing apron, and has a steep upstream slope, and a 1:1 downstream slope. The sill produced a well-formed hydraulic jump, good energy dissipation, self cleaning, and minimal downstream abrasion. A 50-ft-long secondary stilling basin is placed at El. 1285 ft to prevent erosion from residual turbulent energy. It also minimizes the required downstream river channel protection. Riprap design for the immediate downstream river channel was developed using Configuration No. 7.

Constructing a stepped spillway on the downstream slope of the existing spillway, will dissipate little energy. Steps did not dissipate enough energy to retain the hydraulic jump on the existing apron. This could be combined with other methods studied, but the cost would probably not be economically viable.

Downstream riprap guidelines are provided. The average river velocity may still create erosion downstream of the diversion dam structures. If riprap is used downstream of the secondary stilling basin, it should have D_{50} of 22.2 inches in diameter, minimum thickness of 30 inches, 35 ft long, and slopes down to El. 1265 ft or bed rock. As an alternative to riprap, a 20-ft-deep cutoff wall may be used.

Hydraulic Modeling Preparation

Model Construction

The Granite Reef Diversion Dam sectional or two-dimensional model was studied in a laboratory flume that was 36-3/16 inches-wide. A 1:30 geometric scale was chosen to maximize the height of the model within the flume facility. This scale allowed for the projected reservoir head required for 200,000 ft^3/s and for a 14 ft drop below the existing downstream apron elevation. The construction was mainly marine quality plywood and high density urethane.

Figure 3 shows the dimensions of the pertinent features of the model, as initially constructed. Upstream of the dam, 120 ft of the silted forebay was modeled. The flow transitions from the floor of the flume to the modeled reservoir silt level on a 240-ft ramp. The ogee crest and apron are modeled after the April 9, 1917 drawings. The existing 75 ft downstream apron was modeled just downstream from the spillway. Three removable apron sections downstream of the existing apron provided flexibility in the apron study.

The sectional model provided good results, assuming that flow over the diversion dam is uniform across the entire width of the river channel and no influence is anticipated from the sluiceway flows, topography, and rock outcropping within the spillway, and the flash gates on the crest of the dam have equally overturned**.**

Similitude

Froude law relationships were used in this study to ensure the fluid's dynamic similarity. The Froude number was chosen because the hydraulic performance of the model and prototype structures are primarily dependent on gravitational and inertial forces. The Froude number[1] is:

$$F = \frac{V}{\sqrt{Lg}}$$

where V is the velocity, L is the characteristic length, and g is the acceleration of gravity. Froude similitude is achieved by setting the model's Froude number, F_m, equal to the prototype's Froude, F_p, number, such that

$$\frac{F_m}{F_p} = \frac{V_m}{V_p}\sqrt{\frac{g_p L_p}{g_m L_m}} = 1$$

where, $_m$ refers to the model and $_p$ refers to the prototype. The 1:30 scale model of Granite Reef Diversion Dam has the following scaling relations:

Length ratio:

$$L_r = L_m / L_p = 1/30$$

Velocity ratio:

$$V_r = L_r^{1/2} = (1/30)^{1/2} = 1/5.477$$

Discharge ratio

$$Q_r = L_r^{5/2} = (1/30)^{5/2} = 1/4929.5$$

Time ratio:

$$T_r = L_r^{1/2} = (1/30)^{1/2} = 1/5.477$$

These ratios are used by multiplying the prototype value by the appropriate ratio to obtain the model value. For example, the prototype discharge of 200,000 ft^3/s is scaled to a 1:30 Froude scale model discharge by

$$\frac{200{,}000\, ft^3/s}{4929.5} = 40.57\ ft^3/s$$

Because the model was limited to 90.45 ft of the 1,000-ft spillway, this result needs to be multiplied by the modeled crest width to total crest width ratio (90.45/1000) which gives 3.670 ft^3/s.

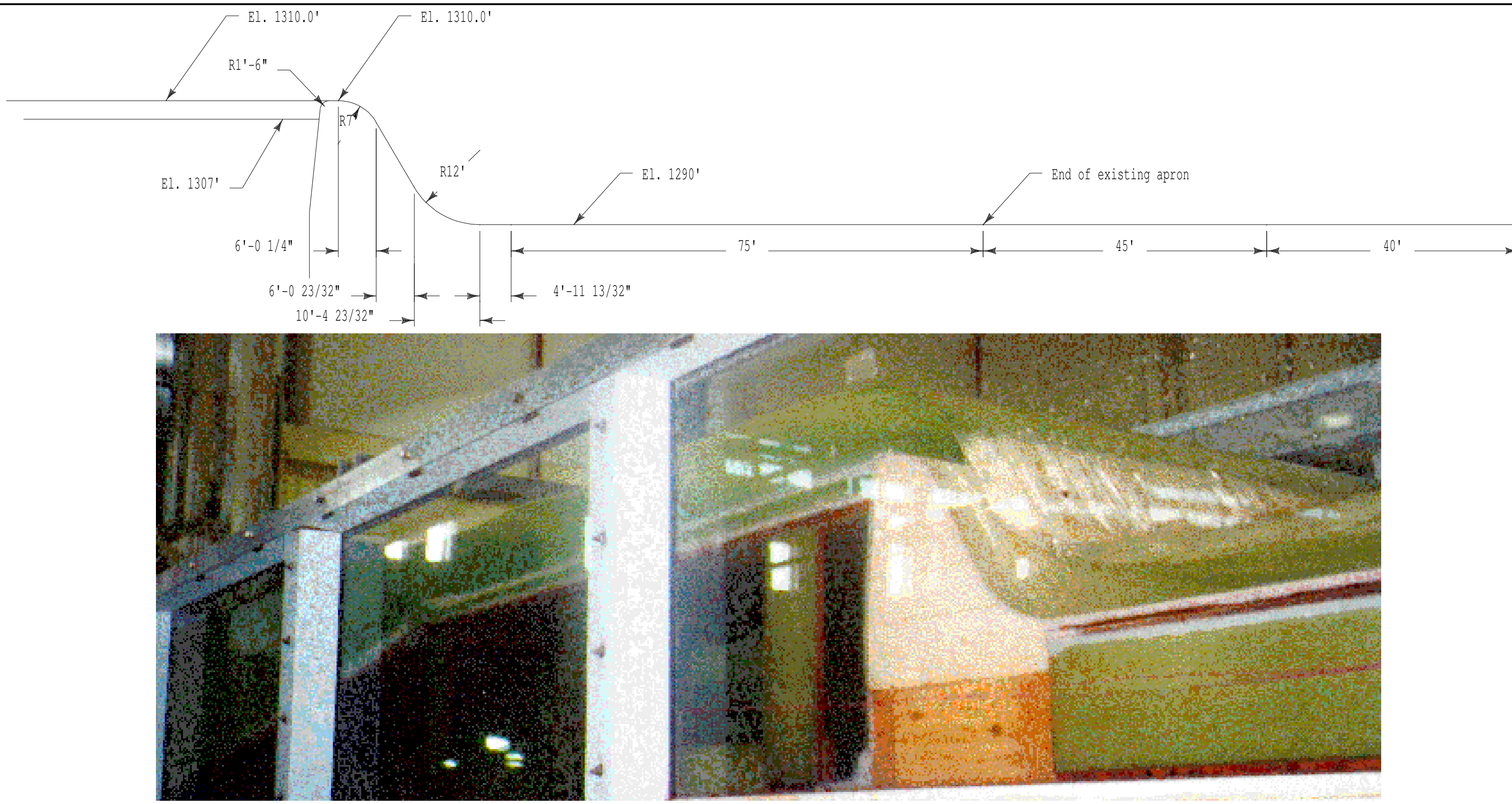


Figure 3. Granite Reef Diversion Dam model. *Prototype dimensions are shown. The photograph shows the forebay is partially silted (El. 1307 ft). The forebay has been constructed to model two different silted conditions: fully silted (El. 1310 ft) and partially silted (El. 1307 ft).*

Tailwater Stage-Discharge Relationship

In March 1982, backwater and scour studies[2] was performed from river cross sections surveyed in 1980. The studies were performed for the design of a scour control structure for the inverted siphon that is approximately 400 ft downstream from Granite Reef Diversion Dam. In 1990, a cross section at the scour control structure was resurveyed and a backwater study performed to determine existing conditions. The tailwater stage-discharge relationship at the scour control structure are shown in Table 2. It was assumed that the tailwater for the diversion dam was the same as stage-discharge relationship at the scour control structure.

The four stage-discharge relationships used in this model study to analyze the hydraulic jump characteristics are shown in Table 1. The values from the two studies and the values used in this study are plotted in Figure 4. These assumptions were verified using photographs of the dam operating under know discharges

Table 2. Stage-discharge relationship for the scour control structure. The report year from which each row of data came from, is indicated in the first column.

Study	Total discharge (ft^3/s)	Stage (ft)
1990	80,000	1298.9
1990	120,000	1302.0
1990	190,000	1306.6
1982	245,000	1309.5
1982	290,000	1311.9
1982	360,000	1315.2

Table 1. Tailwater stage-discharge values used in model study.

Total discharge (ft^3/s)	Stage (ft)
50,000	1296.5
100,000	1300.5
150,000	1304.0
200,000	1307.2

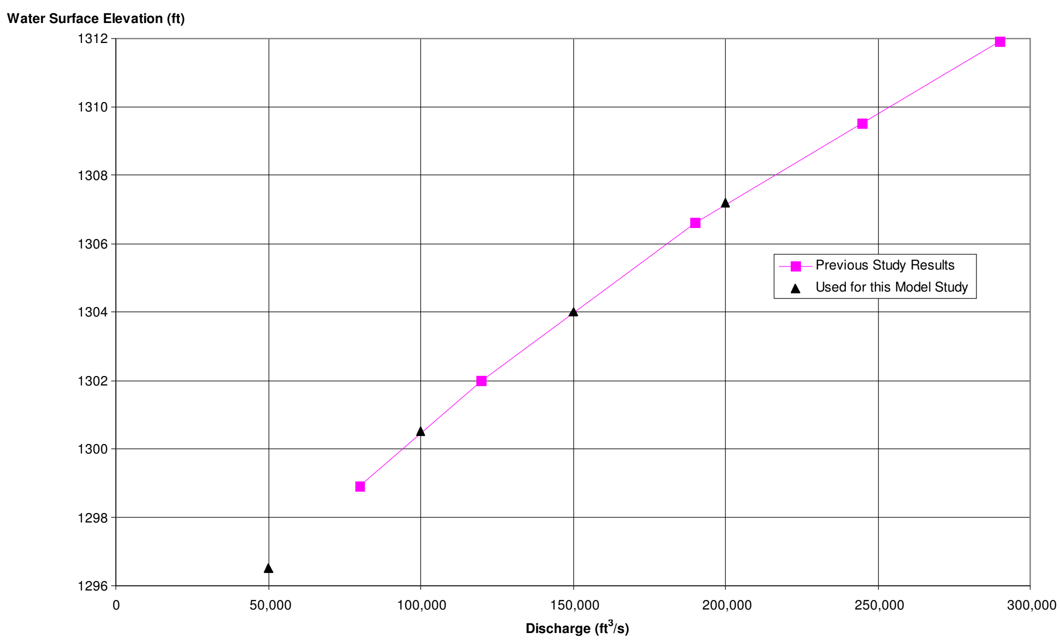


Figure 4. Tailwater stage-discharge relationship. *The result of the March 1982 and 1990 studies are plotted with the values used for this study.*

Investigation

Ogee Crest Stage-Discharge Relationship

To determine the reservoir stage-discharge relationship of the 1,000-ft-wide ogee crest, two fixed surfaces were used to model a fully silted forebay and a partially silted forebay. The fully silted forebay used a flat fixed surface at El. 1310 ft. The partially silted forebay modeled a flat fixed surface at El. 1307 ft. Since this was a sectional model, three-dimensional effects caused by upstream river bends, variations in depth, localized deposition of sediment, and non-uniform overturning of flash gates are ignored. The stage-discharge relationship is independent of downstream conditions due to supercritical flow on the spillway.

The model with fully silted forebay displayed characteristics of a broad-crested weir. This included a Froude number near one (near critical flow) over the silted portion of the forebay and the associated standing waves that are common to flow approaching critical state. The amplitude, size, and positions varied greatly with change of discharge. Results of these tests are presented in Table 3, Figure 5, Figure 6, and Figure 7.

Table 3. Stage at south stilling well to discharge relationship for fully silted forebay.

Prototype discharge (ft^3/s)	Reservoir elevation at south stilling well (ft)
25,000	1314.6
50,000	1317.1
75,000	1319.1
100,000	1320.7
125,000	1322.4
150,000	1323.8
175,000	1325.1
200,000	1326.3

While modeling a partially silted forebay (El. 1307 ft), standing waves were only noticeable during the higher flows. The results are shown in Table 4, Figure 5, Figure 6, and Figure 8.

SRP currently uses two equations to estimate discharges over the ogee crest. Both are presented in Figure 5 for comparison.

The change in amplitude, size, and positions of the standing waves with respect to discharge, indicates that given any flow, the depth measurements at this diversion dam could widely vary while the forebay is fully or mostly silted. This suggests that the diversion dam is generally not a good measurement device while the forebay is mostly silted. This phenomenon is worse for higher discharges.

Table 4. Stage at south stilling well to discharge relationship for partially silted forebay.

Prototype discharge (ft^3/s)	Reservoir elevation at south stilling well (ft)
25,000	1314.3
50,000	1316.0
75,000	1317.4
100,000	1318.7
125,000	1319.8
150,000	1320.7
175,000	1321.8
200,000	1322.5

It is difficult to quantify how much error is present in the discharges measured with the sectional model. The three dimensional approach flow will cause varying unit discharges across the ogee crest, with the river bend upstream, variations in forebay bottom elevation, and various discharges from the two sluice ways also contributing to the error.

The stage-discharge relationships presented should provide values that are in the middle of the ranges that will likely be encountered for these silted conditions.

The best fit lines for the fully silted and partially silted conditions shown in Figure 7 were generated from a linear regression of the head to the apparent discharge coefficient. These are used with the standard weir equation, $Q = CLH^{3/2}$, where Q is the discharge in ft^3/s, C is the discharge coefficient, and H is the applied head in ft. For the fully silted condition, the discharge $Q = (3.13+0.0295H)(1000)H^{3/2}$ and is shown in Figure 5 The partially silted condition uses $Q = (2.13+0.196H)(1000)H^{3/2}$.

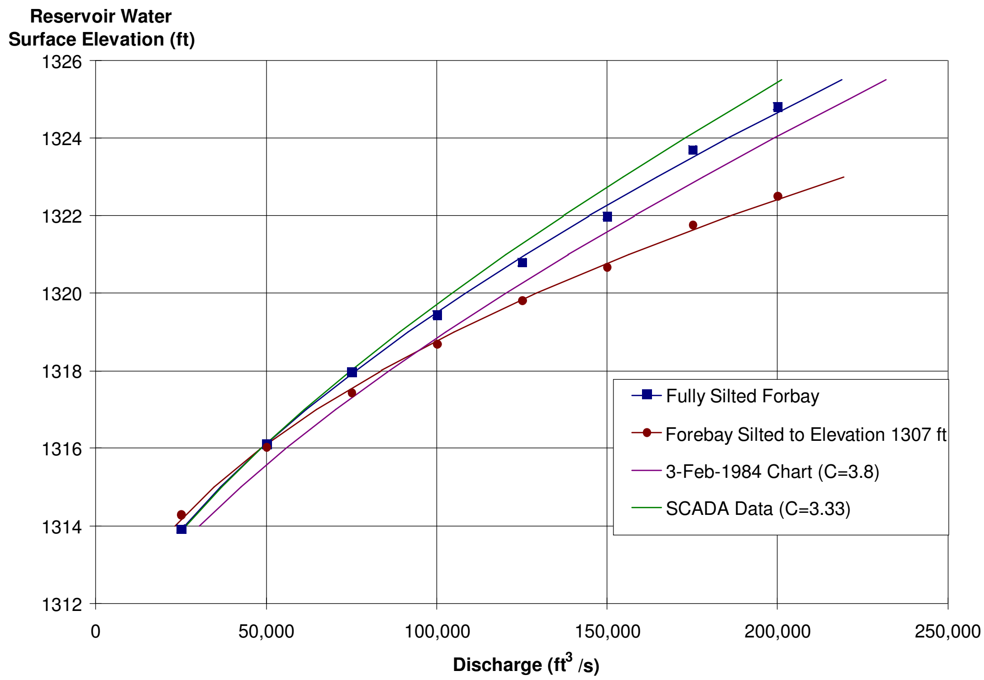


Figure 5. Stage discharge relationship of the ogee crest from laboratory results and discharge equations currently used by SRP. *Since the fully silted forebay behaved like a broad crested weir, interpolating discharge relationship for silted conditions below 1307 ft is inadvisable.*

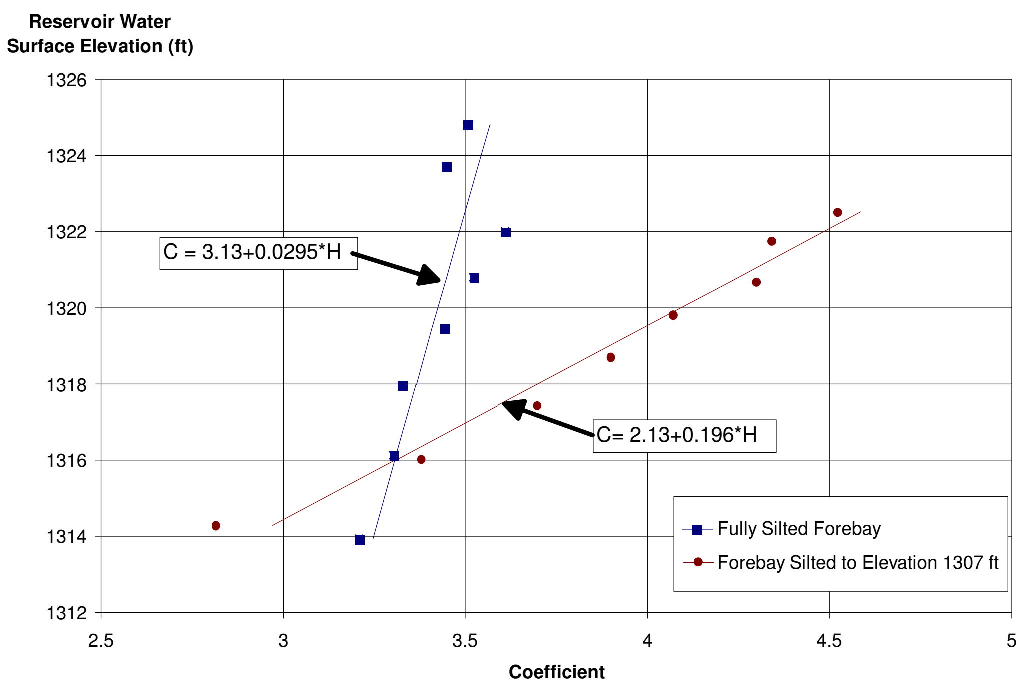


Figure 6. Discharge coefficients for the ogee crest for both silted conditions**.** *The equations were generated from a linear regression of the head to the apparent discharge coefficient.*

Figure 7. Fully silted forebay with prototype discharge of 200,000 ft^3/s. *The standing waves are typical of flows that have a Froude number close to 1.* $F_r=0.53$.

Figure 8. Partially silted forebay with prototype discharge of 200,000 ft^3/s. *Standing waves are not as prevalent as with the fully silted condition.* $F_r=0.58$.

Existing Spillway Apron Length Extension

This portion of the study was to determine the minimum apron length required to retain the hydraulic jump on the apron with existing tailwater. Tests were conducted for discharges of 50,000, 100,000, 150,000, and 200,000 ft^3/s. It was assumed that friction losses would increase with apron length until enough energy was lost that the hydraulic jump would form.

The configuration of the model during this test is shown in Figure 3 and Figure 9 with the forebay level modeled at 1307 ft. The procedure to determine the apron length included raising the tailwater to the elevation indicated by Table 1, and documenting the location of the hydraulic jump. For discharges of 50,000, 100,000, 150,000, and 200,000 ft^3/s, the required length to the upstream toe of the hydraulic jump was equal to or greater than 180 ft downstream from the toe of the dam. It was determined that the jump was likely being forced by the sudden drop at the end of the 240 ft long section.

Testing determined that the position of the jump was very sensitive to the tailwater stage. It was noted that for a discharge of 200,000 ft^3/s, a jump would form at the base of the dam if the tailwater elevation was at or above 1310.9 ft. This implies that if an additional 3.7 ft of tailwater were provided, the jump would be forced at the base of the dam for the existing 75 ft long apron.

The required apron length appears to be longer than what would be feasible.

Figure 9. Looking downstream at the extended apron. *The hydraulic jump appears to be forced by the sudden drop at the end of the model.*

Sill and Secondary Stilling Basin

Sills to force a hydraulic jump on the existing apron, were investigated because the apron length study did not provide favorable results. All testing was performed with the forebay silted to El. 1307 ft, because that configuration created higher velocities down the spillway slope.

Once testing began, it became apparent that a sill at any location on the existing apron could not provide enough energy dissipation by itself. This was due to the difference in elevation between the existing apron (1290 ft) and the thalweg of the river (about 1285 ft). This 5-ft energy head and the head from having a 3.75-ft to 5.625-ft high sill required additional energy dissipation. Secondary stilling basins of varying lengths and elevations were also examined during these tests to dissipate the energy from the high velocity eddies and jets downstream of the sill.

Several different sill and apron configurations were tested. The different configurations tested were eliminated if they did not fit the following criteria:

- The configuration forms a good hydraulic jump upstream of the sill.
- The configuration has the lowest average bottom velocities downstream of the secondary stilling basin that will minimize size and cost of the secondary stilling basin and downstream river channel protection.
- The configuration was not sensitive to tailwater elevation.
- The configuration has self-cleaning characteristics upstream of the sill, with minimal abrasion and sediment impact on the upstream slope.
- The configuration has a minimal amount of sediment trapped in the area immediately downstream of the sill, and minimal abrasion on the backside of the sill and apron.

Consideration was also given to the fact that a sill built at the end of the existing apron may be less expensive to construct due to the ability to tie into the existing cutoff wall. The thickness of the concrete apron upstream of the cutoff wall is unknown across the width of the diversion dam. An additional goal of the study was to determine an economically feasible solution. All sill and stilling basin combinations tested were examined with constructiablitly and cost considered.

Seven of the tested sill and secondary stilling basin configurations are presented in this report. Most of the configurations were only tested to the point where one or more of the criteria were not satisfied.

Sill Configuration No. 1

Sill configuration No. 1 is shown in Figure 10. For this configuration, three sills with varying heights at the end of the existing concrete apron were tested. Sills tested had heights of 1.875-ft, 3.75-ft, and 5.625-ft. Each had vertical upstream and downstream faces.

The 1.875-ft sill did not retain the hydraulic jump on the upstream apron at the proper tailwater elevation of 1307.2 ft. This sill was tested no further.

The 3.75-ft sill performed well with good energy dissipation at the predicted tailwater; however, at 200,000 ft^3/s it swept out if the tailwater dropped 3.75-ft lower than the estimated elevation. Description of the performance of the 3.75-ft sill as the tailwater is reduces is given in Table 5. During this test, a force of 4809 lb/ft-length was measured on the upstream surface for discharge of 200,000 ft^3/s.

Table 5. Description of 3.75-ft-high sill performance with varying tailwater.

Tailwater Elevation (ft)	Description
1307.2	Jump ponding against toe of spillway.
1306.3	Beginning of jump at toe of spillway, rough tailwater.
1305.5	Beginning of jump 18 ft downstream from the toe of spillway. Rough tailwater
1304.9	Beginning of jump 33 ft downstream from toe of spillway. Smooth tailwater.
1303.5	Jump swept off apron.

The 5.625-ft sill maintained the hydraulic jump on the apron for any tailwater elevation, however, the downstream water surface was extremely rough.

In the case of the 3.75 ft and 5.625-ft sills, additional protection would be needed for erosion prevention downstream of the sill. The flow plunging off these sills created high-velocity jets that would impact the downstream surface and require a secondary stilling basin 75 to 100 ft long with riprap beyond that. These configurations were determined unacceptable, due to the inability to clean the sediment from the upstream area of the sill, and the amount of additional protection required downstream.

Sill Configuration No. 2

Sill configuration No. 2 is shown in Figure 11, and was designed to improve sediment cleaning in the upstream area of the sill. It was a 5-ft-high sill with a 1:1 upstream slope.

For a discharge of 50,000 ft^3/s, the hydraulic jump was well formed, however the sill did not self clean. For discharges between 100,000 ft^3/s and 200,000 ft^3/s, and with normal tailwater, there are two stable conditions for the first stilling basin. Either the hydraulic jump was stable on the first stilling basin (desirable), or the jump was swept out and was stable in the secondary basin. The latter of the two was the case shown in the photograph in Figure 11, and produces undesirable high velocity jets in the downstream of the river channel.

For tailwater elevations that corresponds to 100,000 ft^3/s, 150,000 ft^3/s and 200,000 ft^3/s, and with 10 percent additional discharge, the hydraulic jump was swept out of the basin. This configuration was determined unacceptable due the sensitivity of the hydraulic jump to the tailwater elevation and the possibility that three-dimensional effects would cause sweep out of the jump.

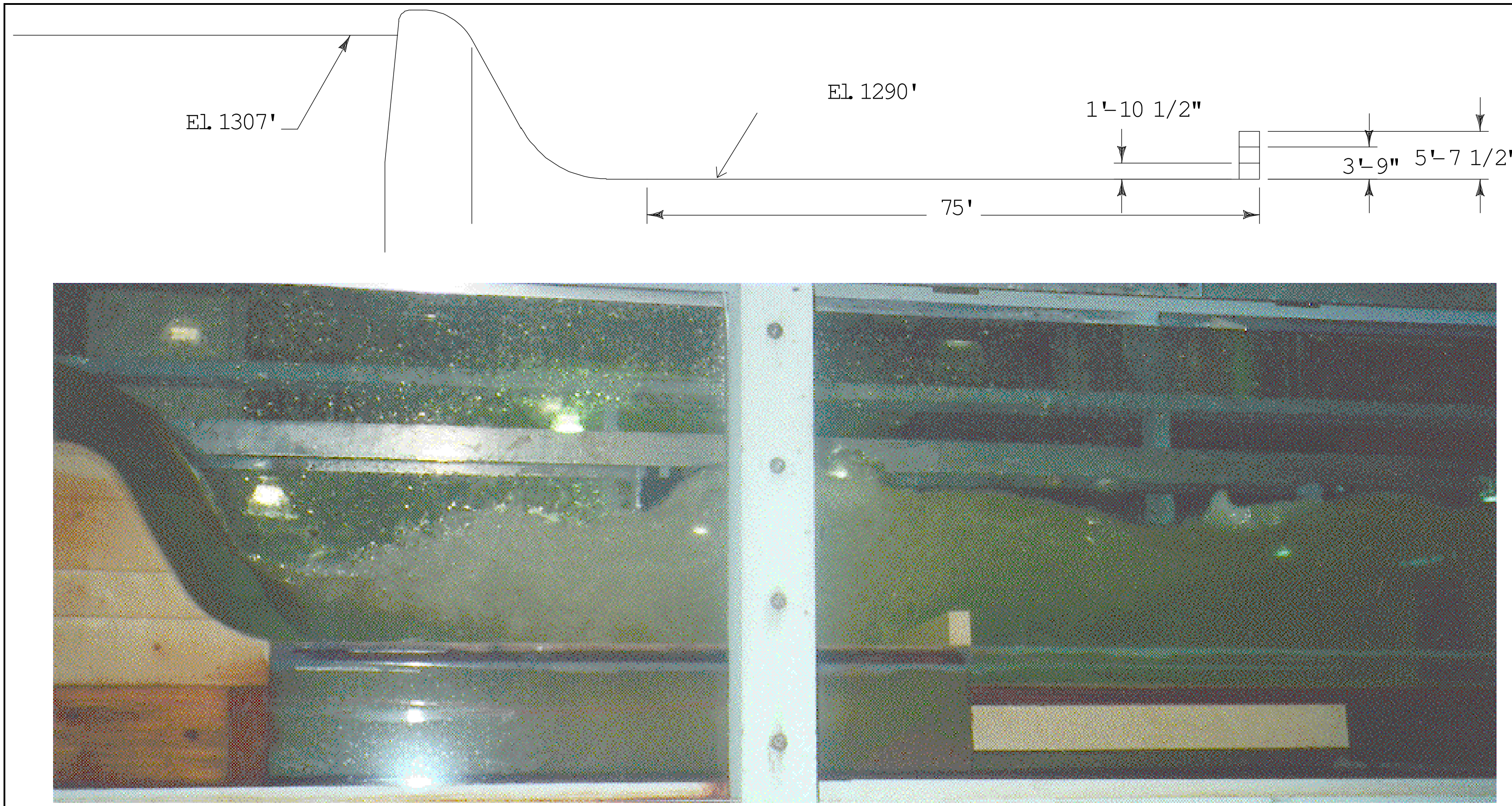
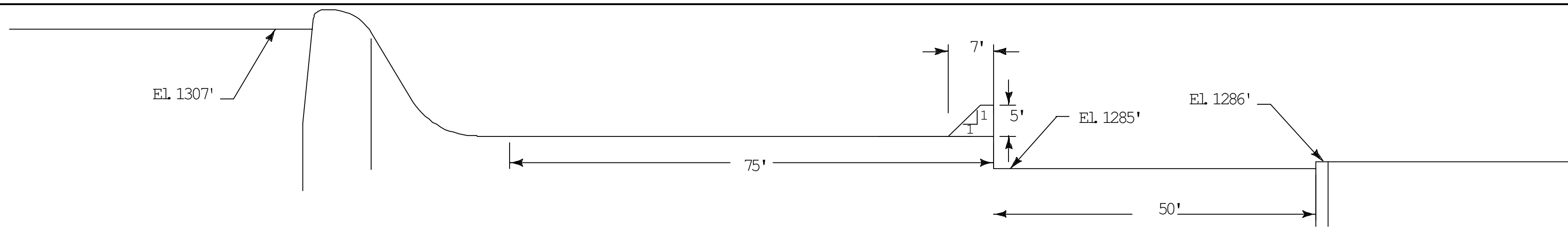


Figure 10. Sill configuration No. 1. *This configuration was tested with three different sill heights, 1.875-ft, 3.75-ft, and 5.625-ft. The hydraulic jump swept out for the 1.875-ft sill. The 3.75-ft sill was determined best for minimizing downstream waves, but did not clean sediment from the upstream area of the sill, because the jet does not imping on the sill.*



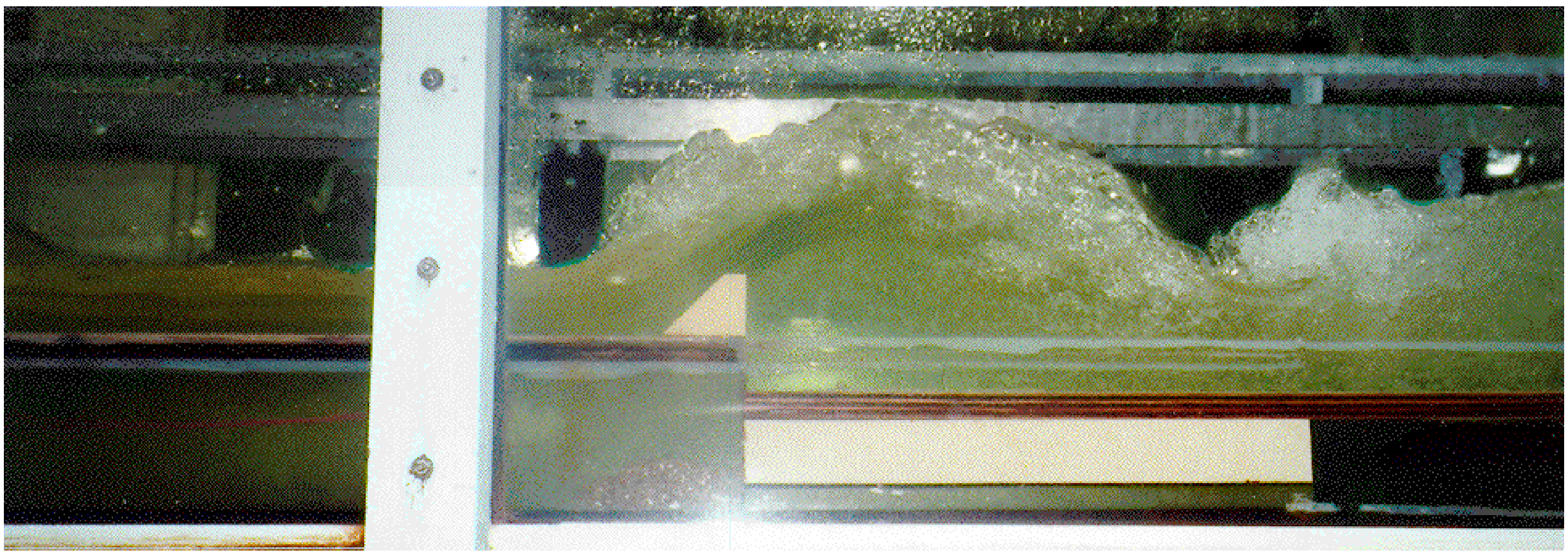


Figure 11. Sill configuration No. 2. *The hydraulic jump can sweep out for discharges from 100,000 to 200,000 ft^3/s and normal tailwater. The photograph shows a discharge of 200,000 ft^3/s, normal tailwater, and the swept out hydraulic jump.*

Sill Configuration No. 3

Sill configuration No. 3 is shown in Figure 12. This configuration had the sill 25 ft upstream of the end of the apron, and used steep upstream and downstream slopes. This configuration did not clean the sediment from the area upstream of the sill. It also caused plunging into the secondary stilling basin similar to configuration No. 1, which requires a longer secondary stilling basin. It also did not have the construction advantage of configuration No. 1, which could be constructed by tying into the cutoff wall of the existing apron, thus requiring the more expensive installation. This option did not provide any additional advantages.

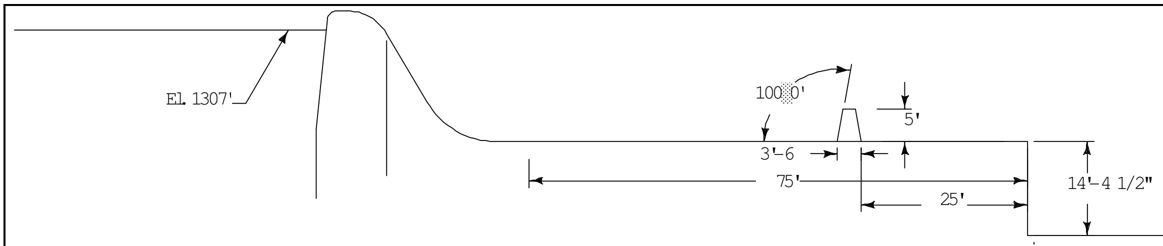


Figure 12. Sill Configuration No. 3. *This sill did not clean sediment from the upstream area of the sill, and caused a plunging flow into the secondary basin similar to configuration No. 1, and would require a longer secondary stilling basin.*

Sill Configuration No. 4

Sill configuration No. 4 is shown in Figure 13. This configuration had the sill 50 ft upstream of the end of the apron, and used steep upstream and downstream slopes. This configuration created a well-formed hydraulic jump at the base of the spillway and good sediment cleaning characteristics in the area upstream of the sill. However, this configuration trapped sediment in the wake zone immediately downstream of the sill, which may cause significant abrasion damage to the existing apron and the sill.

Sill Configuration No. 5

Sill configuration No. 5 is shown in Figure 14. This configuration had the sill 50 ft upstream of the end of the apron, and used a 1:1 upstream slope and a 2:1 downstream slope. This configuration was designed to keep the flow attached to the downstream face of the sill, thus cleaning the area downstream of the sill of sediment. It was also designed to reduce sediment and hydraulic impact on the upstream surface. However, the flow still separated from the leading upper edge of the sill, and the sediment behind the sill was not swept clean. During testing, the sediment worked its way upstream on the downstream face of the sill, which may indicate potential for significant abrasion damage on the downstream and top sill surfaces.

Comparisons of photographs and video clearly showed that the hydraulic jump is not nearly as well formed as the hydraulic jump in configuration No. 4. During this test, a force of 5612 lb/ft-length was measured on the upstream sill surface for a discharge of 200,000 ft^3/s.

Sill Configuration No. 6

Sill configuration No. 6 is shown in Figure 15. This configuration had the sill 50 ft upstream of the end of the apron, and used a 1:1 upstream and 1:1 downstream slopes. This configuration was designed to reduce the amount of concrete used in configuration No. 5. It also retains some sacrificial concrete for the sill, and minimizes the movement of the sediment on the existing apron in the area downstream of the sill. However, the hydraulic jump was not well formed in the area upstream of the sill, and displayed the characteristics of sweeping out with low tailwater similar to configurations No. 2 and 5.

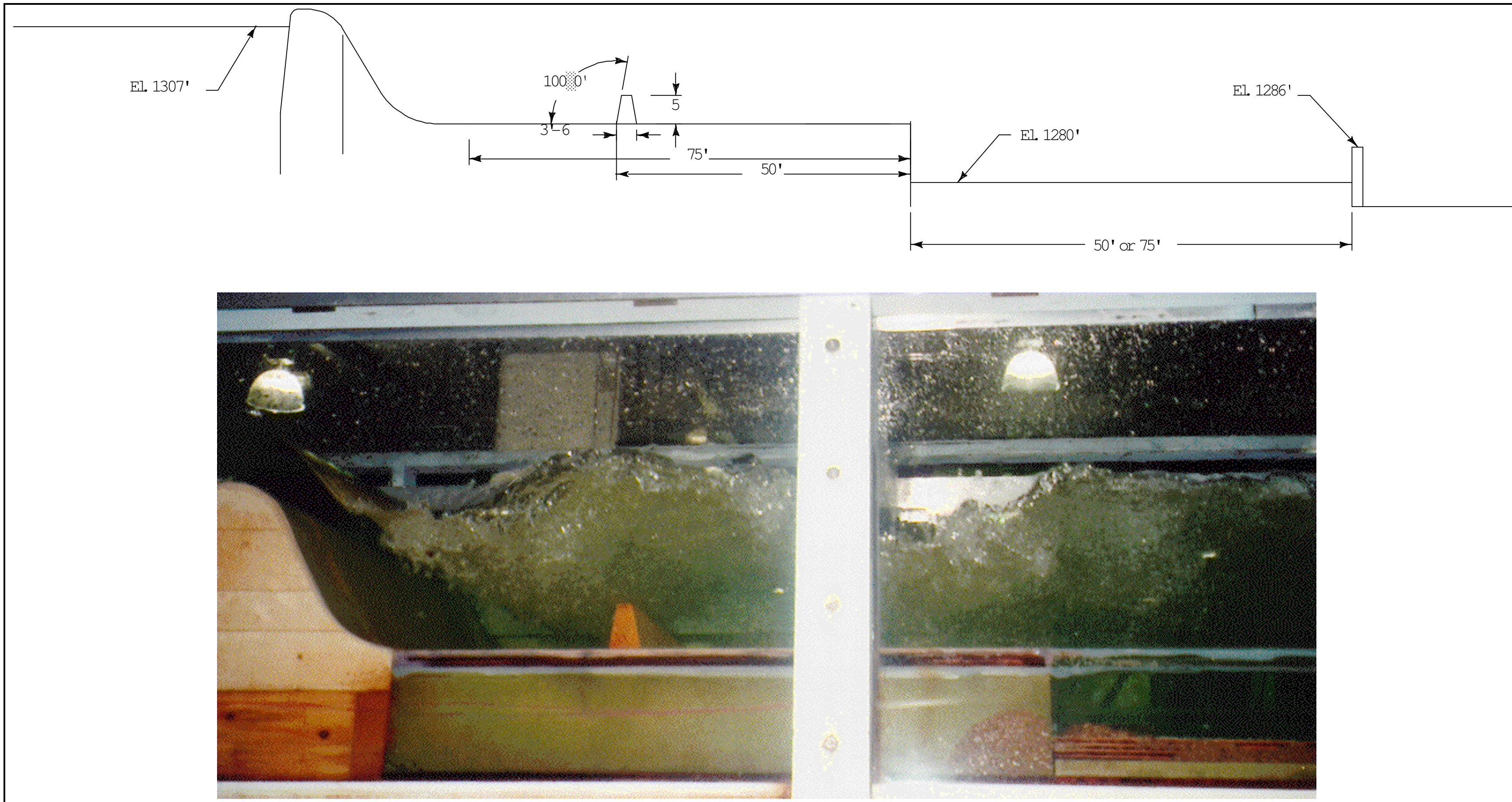


Figure 13. Sill configuration No. 4. *The photograph shows a well-formed hydraulic jump in front of the sill. The area upstream of the sill is self cleaning due to the high impact on the upstream slope of the sill for all flow rates. The flow separates off the upstream edge of the sill and causes a strong recalculation zone behind the sill that will retain typical river sediments.*

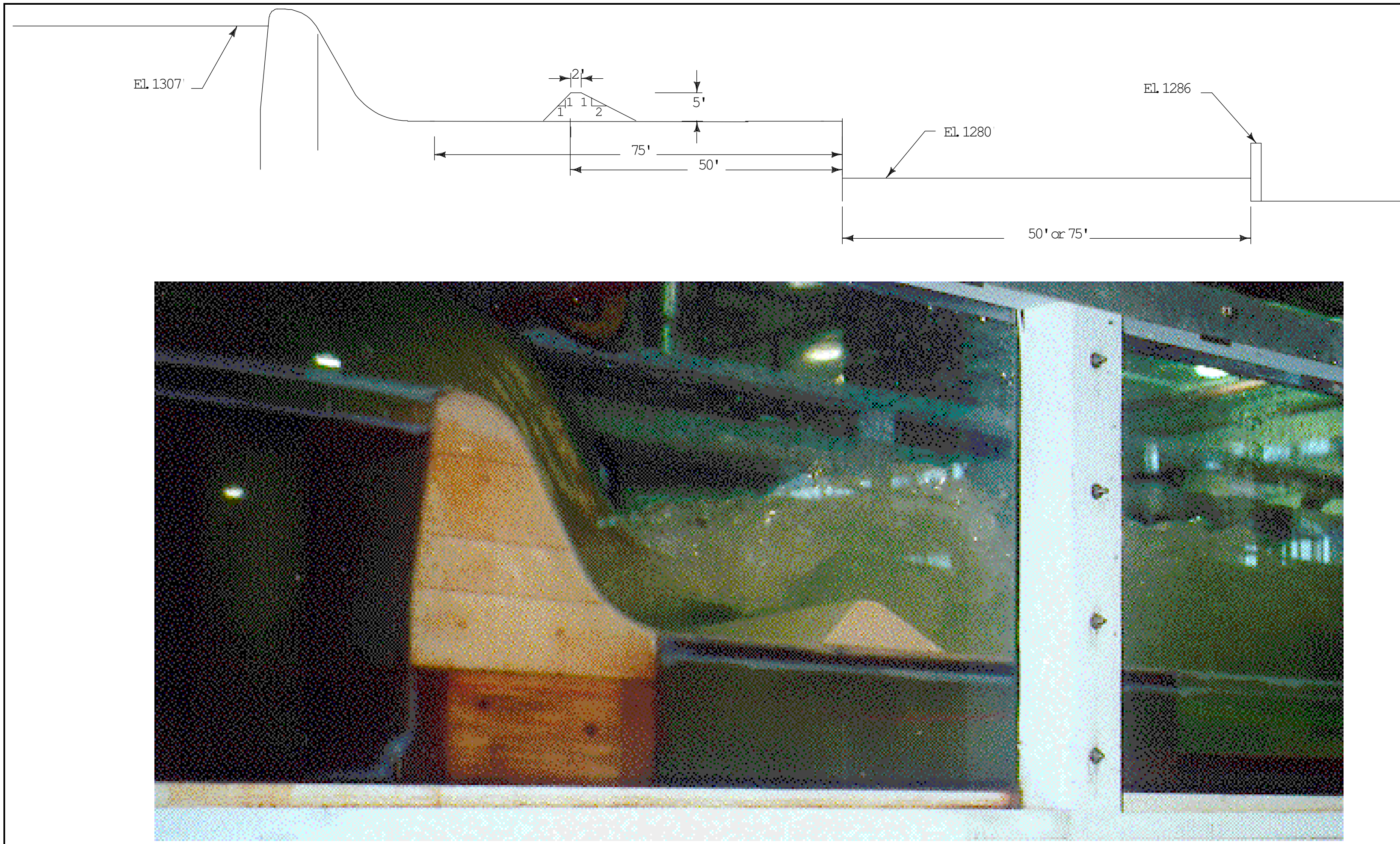


Figure 14. Sill Configuration No. 5. *The photograph shows that the hydraulic jump is not well formed in front of the sill. The area upstream of the sill is self cleaning due to the high impact on the upstream slope of the sill for all flow rates. The flow separates off the upstream edge of the sill and causes a strong recalculation zone behind the sill that will retain typical river sediments. Sediment in the model displayed a lot of movement on the downstream slope and top of the sill. This may indicate a lot of abrasion damage to the downstream and top surfaces.*

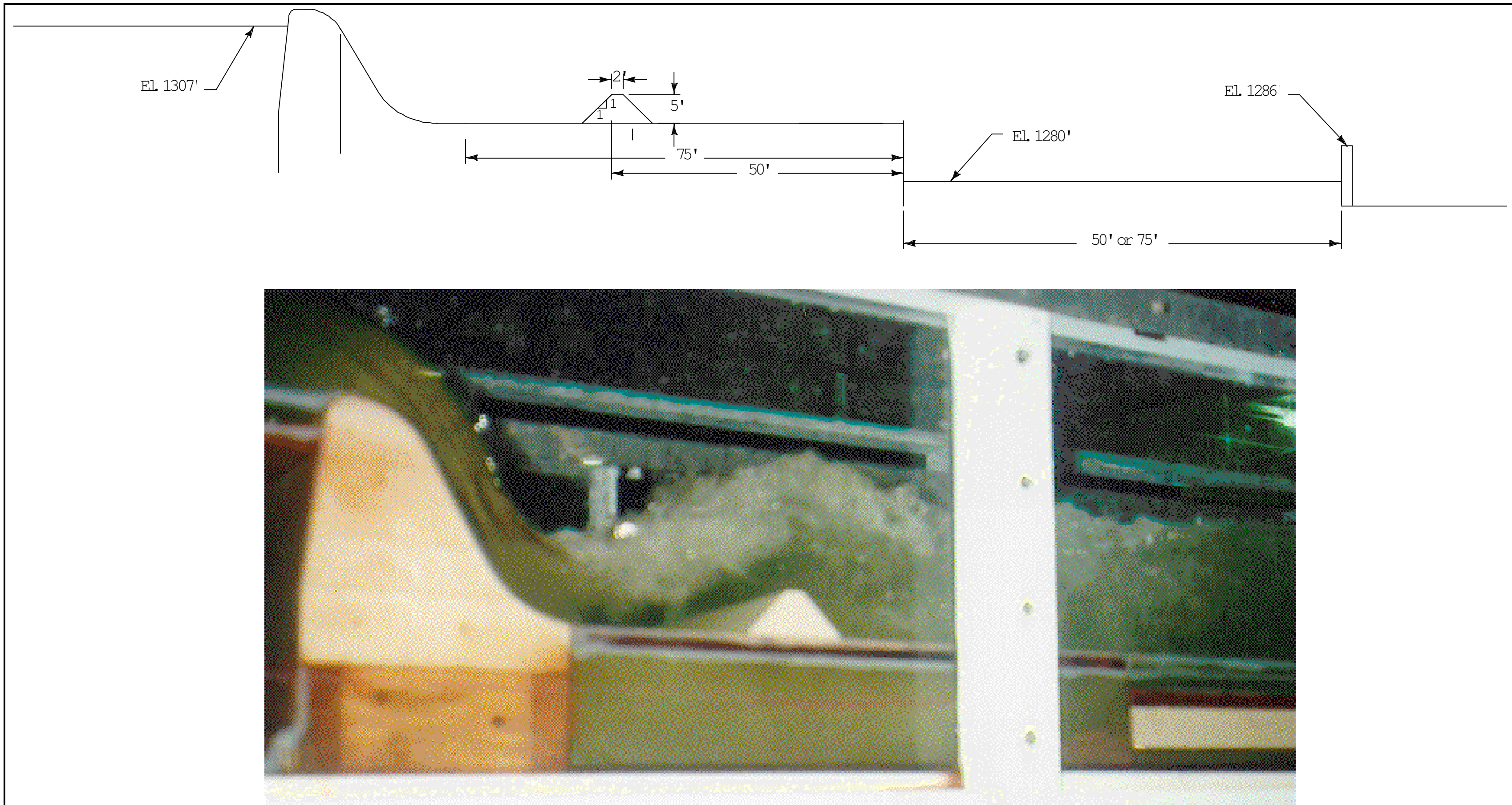


Figure 15. Sill configuration No. 6. *The photograph shows that the hydraulic jump is not well formed in front of the sill, and is not dissipating much energy. For lower tailwater, the jump completely sweeps out for this configuration. Sediment downstream of the sill displayed less movement than in configurations No. 4 and No. 5.*

Configuration No. 7 --Final Design

Sill configuration No. 7 is shown in Figure 16, and is the *preferred configuration* for the sill and secondary basin. The 100º upstream slope was designed after configuration No. 4 to develop a good hydraulic jump. The 1:1 downstream slope was designed after observations from No. 6.

This configuration develops a good hydraulic jump and good sediment cleaning characteristics upstream of the sill for discharges from 50,000 ft^3/s to 200,000 ft^3/s. Some sediment was trapped in the downstream area of the sill, but displayed less movement when compared to other tested configurations.

This configuration maintained a hydraulic jump for discharges up to 250,000 ft^3/s and minimal tailwater. Minimal tailwater describes the condition when the tailwater elevation is at the lowest that can be set in the model, and is restricted only by the secondary basin, and the downstream topography in the model. This indicates that this sill configuration can tolerate large variations in unit discharges over the ogee crest due to three dimensional effects. The downstream three dimensional effects due to variations of river bottom elevation and effects from the sluiceways are unknown, but this option should be minimally sensitive to these effects.

During discharge up to 200,000 ft^3/s, the flow impacts the existing apron just downstream of the sill. The 50-ft-long secondary stilling basin at El. 1285 ft as shown in Figure 16 helps to reduce jet velocities before the flow reaches the downstream riprap section.

The design discussed in Downstream River Channel Protection section of this document was developed using this configuration.

Velocities were measured to help determine the elevation of the secondary stilling basin, and to assist in the design of riprap. The maximum velocities were not significantly lower for the secondary stilling basin at El. 1280 ft. Therefore, the secondary stilling basin at 1285 ft is recommended and all remaining testing was conducted with that configuration.

Providing a maintenance access through the sill was partially investigated for configuration No. 1. What was tested created major downstream turbulence, and this line of investigation was discontinued in favor of investigating better sill designs. If this is to be investigated in the future, it is recommended that a sill with the same profile as the main sill be placed 15 ft downstream from the main sill, and 15 ft of overlap be provided on each side. That investigation may vary sill heights and overlaps lengths.

Stepped Spillway

Another purpose of the study was to investigate a stepped spillway as an option to dissipate energy. Dimensions for the stepped spillway model are shown in Figure 17. The design was developed with consideration of maximizing step height and minimizing the volume of concrete.

Very little energy was dissipated by flow over the steps. (Figure 18). This is indicated by similar depths on the existing apron, and similar hydraulic jumps at the drop-off at the end of the existing apron.

It was determined that the 10.7-ft overtopping head (critical flow on top of crest for 200,000 ft^3/s) was too great for the 20-ft drop to dissipate much energy.

Additional appurtenances with the steps could be used to force the jump on the existing apron, but little benefit and high cost were expected from using the steps, so this option was abandoned.

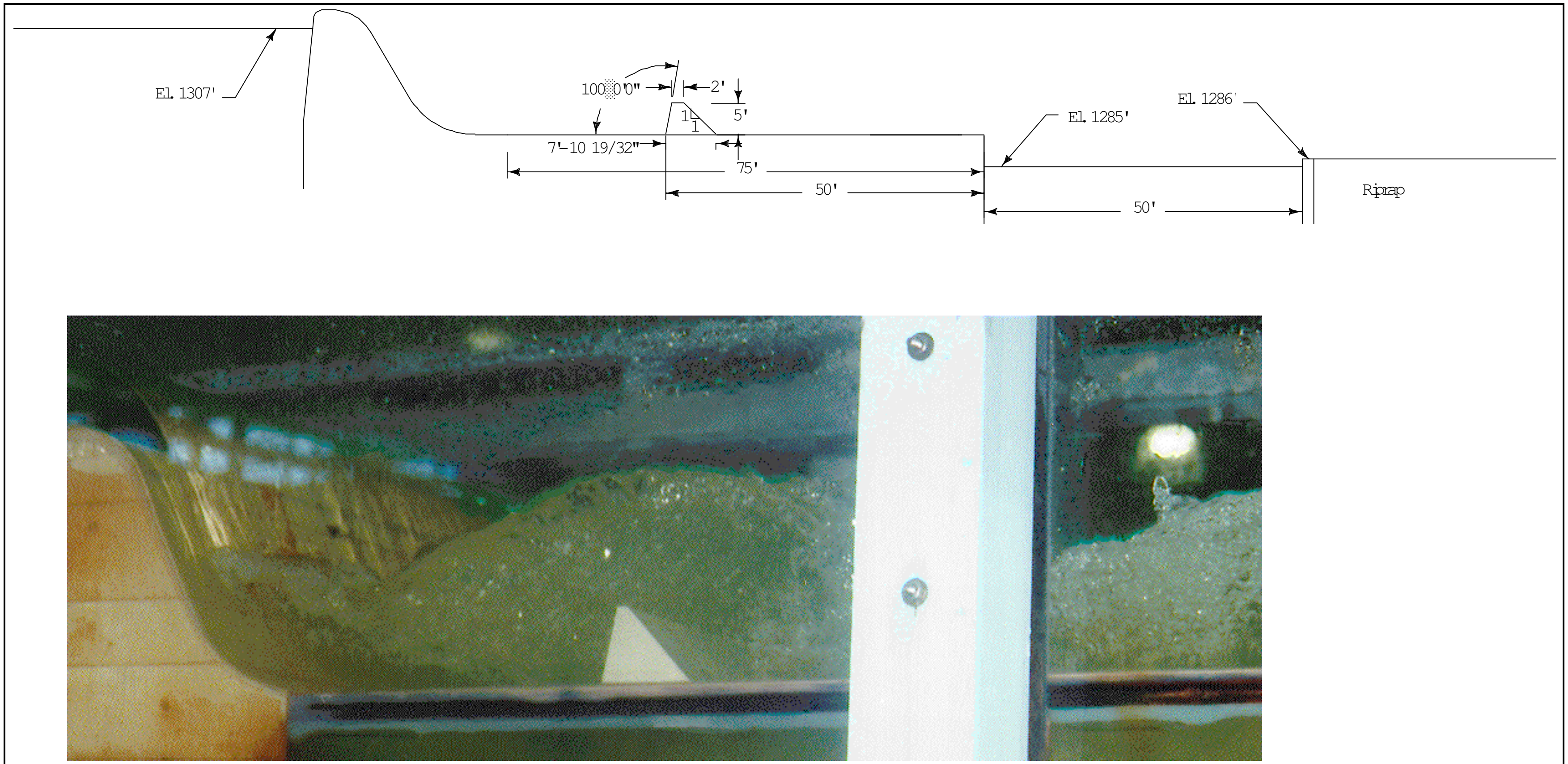


Figure 16. Sill configuration No. 7. *This is the recommended configuration. The photograph was taken with minimal tailwater to show that the hydraulic jump will not sweep out under any condition for discharges up to 200,000 ft^3/s. It appears that a 1:1 downstream slope may be best for this option. This has a similar wake condition downstream of the sill, but allows some sacrificial concrete compared to a vertical downstream face in case severe abrasion Sediment downstream of the sill displayed less movement than in configurations No. 4 and No. 5.*

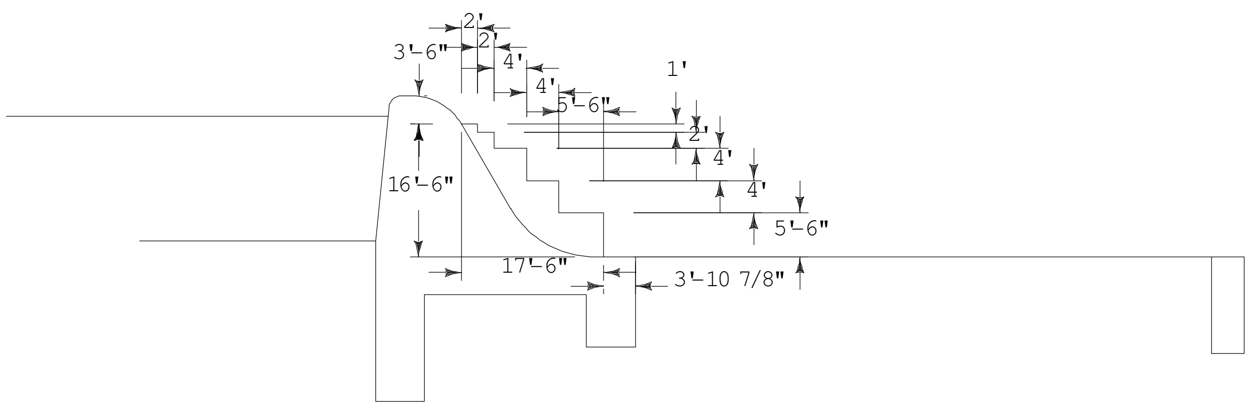


Figure 17 Prototype dimensions of modeled stepped spillway.

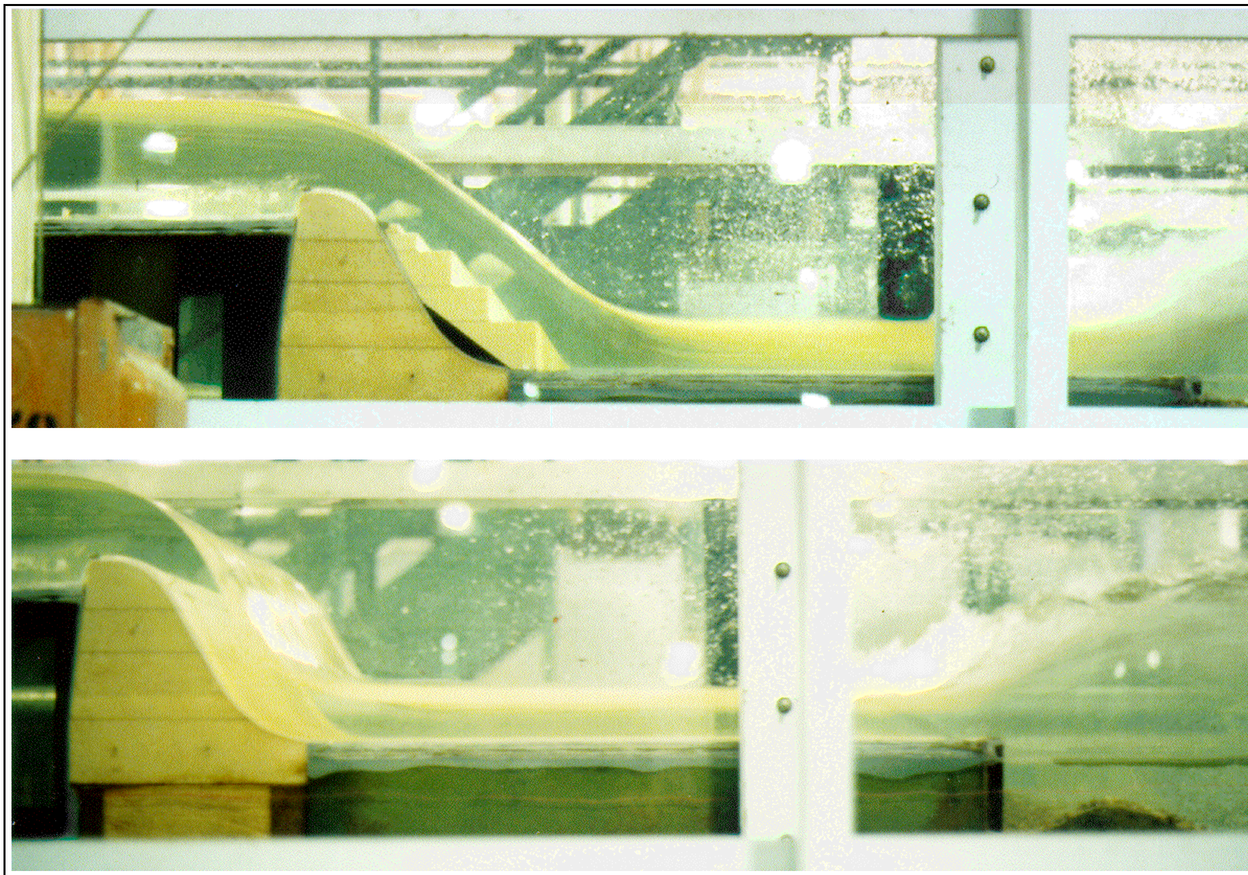


Figure 18. Side view of model stepped spillway and existing condition. *It appears that extremely little energy loss can be attributed to the steps, which is indicated by similar depths on the existing apron, and similar hydraulic jumps at the drop off at the end of the existing spillway.*

Downstream River Channel Protection

Velocities were measured near the bottom of the channel for the preferred option. Velocities immediately downstream of the secondary basin and 80 ft further downstream are shown in

Figure 19 and Figure 20. Velocity data is presented in Table 6, and shows a decrease in bottom velocity caused by a minor flow separation off the sill of the secondary basin. The turbulence from the basin is settled out 80 ft downstream, where the maximum bottom velocities were recorded.

Table 6. Bottom velocity data downstream of the secondary stilling basin for 200,000 ft^3/s.

Location downstream from secondary basin	Average Velocity, (ft/s)	1 Standard Deviation (ft/s)	Average Velocity + 1 Standard Deviation (ft/s)	Average Velocity + 2 Standard Deviations (ft/s)
Immediately downstream	6.07	1.94	8.01	9.95
80 ft downstream	8.76	1.32	10.08	11.4

If the riprap is designed using the average bottom velocity at 80 ft downstream plus two standard deviations, the

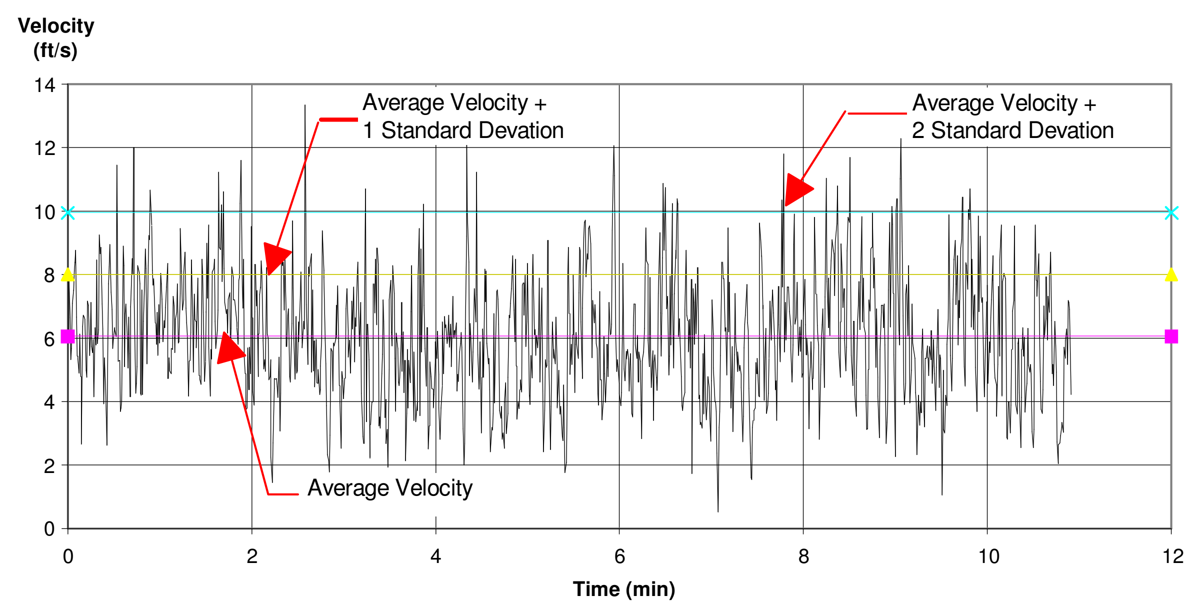


Figure 19. Total velocity near the bottom, immediately downstream from secondary stilling basin.. *Frequency and amount of time above the design velocity should be considered when selecting the design velocity.*

design velocity is 11.4 ft/s. The average velocity calculated in the downstream river channel for 200,000 ft^3/s is estimated to be about 9.4 ft/s. A conservative estimate of the turbulence near the bottom is attained from Figure 20. Adding two standard deviations to this calculated average velocity produces a design velocity of 12 ft/s.

Using 12 ft/s, the U. S. Bureau of Reclamation's [3] riprap sizing criteria, the D_{50} of the riprap should be 21 inches. Using the U. S. Corps of Engineers'[4] criteria, D_{50} of the riprap should be 22.2 inches. This should be wellgraded material with $D_{50}/D_{20} \approx 2$ and $D_{100}/D_{50} \approx 2$. The riprap layer should be the greater of $2D_{50}$ or $1.5D_{100}$. The riprap must be placed over well graded filter material that is not less than half the thickness of the riprap layer. The riprap

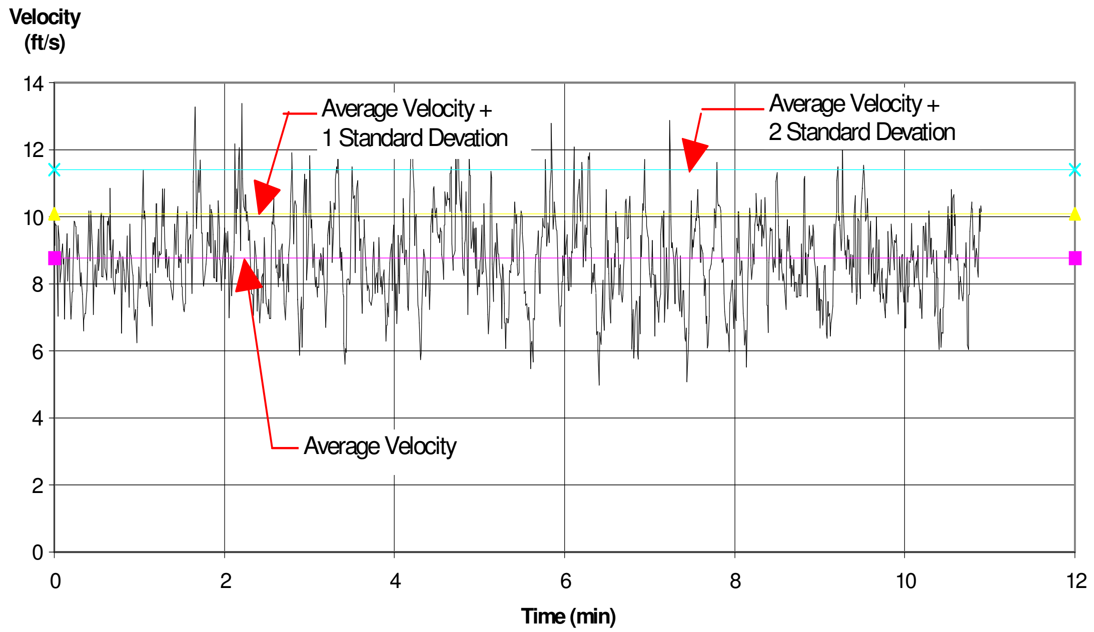


Figure 20. Total velocity near bottom of channel, 80 ft downstream from secondary stilling basin. *This represents near maximum average velocity found in the model. Frequency and amount of time above the design velocity should be considered when selecting the design velocity.*

should extend a reasonable distance downstream of the secondary stilling basin to reduce further erosion of river materials and to prevent undermining of the protection.

The river channel will continue to erode simply due to the small size of the river bed material and the velocities produced by floods. If riprap is used downstream of the secondary stilling basin, it should have a minimum downstream extent of 35 ft. It should slope downward to El. 1265 ft or bedrock. Weight of the rock was assumed to be 165 lb/ft^3. Larger rock will be needed if a rock of lesser weight is used.

About a 20 ft deep concrete cutoff wall placed at the end of the secondary stilling basin could also be used in place of an extensive amount riprap protection. The construction method utilized is entirely up to SRP.

In general, using the maximum recorded turbulence level is a conservative approach to riprap design and should account for most three dimensional effects.

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[1] *Hydraulic Laboratory Techniques*, p. 51, United States Government Printing Office, Denver, 1980.
[2] U. S. Bureau of Reclamation Memorandum: Scour Control Structure -- Salt River Siphon, Granite Reef Aqueduct, Central Arizona Project, Arizona, June 30 1982.
[3] Peterka, A. J. "Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25," pp. 207-221, U. S. Bureau of Reclamation, Denver CO, January 1978
[4] "Hydraulic Design Criteria," US Corps of Engineers, Volume 2, Sheet 712-1, January 1977.